List of Figures

Chapter Thirty-eight

BANK PROTECTION

38-1.0 INTRODUCTION

38-1.01 Purpose

One of the hazards of placing a highway near a river or stream channel or other water body is the potential for erosion of the highway embankment by moving water. If erosion of the highway embankment is to be prevented, bank protection must be anticipated, and the proper type and amount of protection must be provided in the right locations.

Four methods of protecting a highway embankment from bank erosion are available to the designer. These are as follows:

1. relocating the highway away from the stream or water body,
2. moving the water body away from the highway (channel change),
3. changing the direction of the current with training works, and
4. protecting the embankment from erosion.

This Chapter provides procedures for the design of revetments to be used as channel bank protection and channel linings on larger streams and rivers (i.e., those having design discharges generally greater than 1.4 m^3/s). Procedures are also presented for riprap protection at bridge piers and abutments. For small discharges, procedures presented in Chapter Thirty should be used. Emphasis in this Chapter has been placed on rock riprap revetments due to their costs, environmental considerations, flexible characteristics and widespread acceptance. Other channel stabilization methods such as spurs, guide banks retard structures, longitudinal dikes and bulkheads are discussed in *Stream Stability at Highway Structures*, Hydraulic Engineering Circular No. 20.

38-1.02 Erosion Potential

Channel and bank stabilization is essential to the design of any structure affected by the water environment. The identification of the potential for bank erosion, and the subsequent need for stabilization, is best accomplished through observation. A three-level analysis procedure is provided in Hydraulic Engineering Circular No. 20. This procedure is described in this *Manual* in Chapter Thirty. The three-level analysis provides a rigorous procedure for determining the geomorphological characteristics, evaluating the existing conditions through field observations and determining the hydraulic and sediment transport properties of the stream. If sufficient information is obtained at any level of the analysis to solve the problem, the procedure may be stopped without proceeding to the other levels.

Observations provide the most positive indication of erosion potential. Observation comparison can be based on historic information or current site conditions. Aerial photographs, old maps, surveying notes, bridge design files and river survey data available at the INDOT Central Office and at Federal agencies, gaging station records and interviews of long-time residents can provide documentation of any recent and potentially current channel movement or bank instabilities.

In addition, current site conditions can be used to evaluate stability. Even when historic information indicates that a bank has been relatively stable in the past, local conditions may indicate more recent instabilities. Local site conditions which are indicative of instabilities may include tipping and falling of vegetation along the bank, cracks along the bank surface, the presence of slump blocks, fresh vegetation laying in the channel near the channel banks, deflection of channel flows in the direction of the bank due to some recently deposited obstruction or channel course change, fresh vertical face cuts along the bank, locally high velocities along the bank, new bar formation downstream from an eroding bank, local headcuts, pending or recent cutoffs, etc. It is also important to recognize that the presence of any one of these conditions does not in itself indicate an erosion problem; some bank erosion is common in all channels even when the channel is stable.

38-1.03 Symbols and Definitions

To provide consistency within this Chapter and throughout this *Manual*, the symbols in Figure 38-1A will be used. These symbols were selected because of their wide use in many bank and shore protection publications. Where the same symbol is used for more than one definition, the symbol will be defined where it is used.

38-2.0 POLICY

Highway alignments and improvements often cross, encroach upon or otherwise require construction of a new channel or modification of the existing channel. It is necessary to protect the public, the highway investment and the environment from the natural reaction to the highway changes. Department policy requires that the facility, including bank protection, will perform without significant damage or hazard to people and property for flood and flow conditions experienced on a 100-year recurrence interval. The facility, to the maximum extent possible, shall perpetuate natural drainage conditions thus protecting and maintaining the environment.

38-3.0 BANK AND LINING FAILURE MODES

38-3.01 Potential Failures

Prior to designing a bank stabilization scheme, it is important to be aware of common erosion mechanisms and revetment failure modes and the causes or driving forces behind bank erosion processes. Inadequate recognition of potential erosion processes at a particular site may lead to failure of the revetment system.

Many causes of bank erosion and revetment failure have been identified. Some of the more common include abrasion, debris flows, water flow, eddy action, flow acceleration, unsteady flow, freeze/thaw, human actions on the bank, ice, precipitation, waves, toe erosion and subsurface flows. However, it is most often a combination of mechanisms which cause bank and revetment failure, and the actual mechanism or cause is usually difficult to determine. Failures are better classified by failure mode including the following:

1. particle erosion,
2. translational slide,
3. modified slump, and
4. slump.

38-3.02 Particle Erosion

Particle erosion is the most commonly considered erosion mechanism. Particle erosion results when the tractive force exerted by the flowing water exceeds the bank material's ability to resist movement. In addition, if displaced stones are not transported from the eroded area, a mound of displaced rock will develop on the channel bed. This mound has been observed to cause flow concentration along the bank, resulting in further bank erosion.

A special type of particle erosion results in loss of the underlying material resulting in undermining and eventual collapse of the revetment protection. Usually the underlying material is lost through the revetment or piped under the toe of the revetment protection. This failure is very common in and extremely damaging to rigid types of protective linings. Providing a suitable filter, either natural or fabrics in conjunction with hydrostatic relief features, will prevent this failure.

Another frequent type of particle erosion failure occurs at the edges of the protective feature. The interface creates turbulence which in turn increases the tractive stresses placed on the protective layer, underlying layers and the natural bank material beyond the revetment. This failure area needs to receive special attention because extension of the protective feature usually only moves, not eliminates, the failure.

38-3.03 Translation Slide

A translational slide is a failure of riprap caused by the downslope movement of a mass of stones, with the fault line on a horizontal plane. The initial phases of a translational slide are indicated by cracks in the upper part of the riprap bank that extend parallel to the channel. As the slide progresses, the lower part of the riprap separates from the upper part and moves downslope as a homogeneous body. A resulting bulge may appear at the base of the bank if the channel bed is not scoured.

38-3.04 Modified Slump

The failure of riprap referred to as modified slump is the mass movement of material along an internal slip surface within the riprap blanket; the underlying material supporting the riprap does not fail. This type of failure is similar in many respects to the translational slide, but the geometry of the damaged riprap is similar in shape to initial stages of failure caused by particle erosion.

38-3.05 Slump

Slump is a rotational-gravitational movement of material along a surface of rupture that has a concave upward curve. The cause of slump failures is related to shear failure of the underlying base material that supports the riprap revetment. The primary feature of a slump failure is the localized displacement of base material along a slip surface, which is usually caused by excess pore pressure that reduces friction along a fault line in the base material.

38-4.0 REVETMENT TYPES

38-4.01 Common Types

The types of slope protection or revetment commonly used for bank and shore protection and stabilization include the following:

1. rock and rubble riprap,
2. wire-enclosed rock (gabions),
3. pre-formed blocks,
4. grouted rock,
5. grouted fabric,
6. sand/cement bags, and
7. soil cement.

Descriptions of each of these revetment types are included in this Section.

38-4.02 Riprap

Riprap has been described as a layer or facing of rock, dumped or hand-placed to prevent erosion, scour or sloughing of a structure or embankment. Materials other than rock are also referred to as riprap; for example, rubble, broken concrete slabs and preformed concrete shapes (slabs, blocks, rectangular prisms, etc.). These materials are similar to rock in that they can be hand-placed or dumped onto an embankment to form a flexible revetment.

38-4.03 Wire-Enclosed Rock

Wire-enclosed rock (or gabion), revetments consist of rectangular wire mesh baskets filled with rock. These revetments are formed by filling pre-assembled wire baskets with rock and anchoring to the channel bottom or bank. Wire-enclosed rock revetments are generally of two types distinguished by shape: rock and wire mattresses or blocks. In mattress designs, the individual wire mesh units are laid end to end and side to side to form a mattress layer on the channel bed or bank. The gabion baskets comprising the mattress generally have a depth dimension which is much smaller than their width or length. Block gabions, on the other hand, are more equal-dimensional, having depths that are approximately the same as their widths and of the same order of magnitude as their lengths. They are typically rectangular or trapezoidal in shape. Block gabion revetments are formed by stacking the individual gabion blocks in a stepped fashion.

38-4.04 Pre-Cast Concrete Block

Pre-cast concrete block revetments are a recent development. The pre-formed sections which comprise the revetment systems are butted together or joined in some fashion; as such, they form a continuous blanket or mat. The concrete blocks which make up the mats differ in shape and method of articulation but share certain common features. These features include flexibility, rapid installation and provisions for establishment of vegetation within the revetment. The permeable nature of these revetments permits free draining of the bank materials; the flexibility, although limited, allows the mattress to conform to minor changes in the bank geometry. Their limited flexibility, however, subjects them to undermining in environments characterized by large and relatively rapid fluctuations in the surface elevation of the channel bed and/or bank. Unlike wire-enclosed rock, the open nature of the pre-cast concrete blocks does promote volunteering of vegetation within the revetment.

38-4.05 Grouted Riprap

Grouted revetment riprap consists of rock slope-protection having voids filled with concrete grout to form a monolithic armor. Grouted riprap is a rigid revetment; it will not conform to changes in the bank geometry due to settlement. As with other monolithic revetments, grouted riprap is particularly susceptible to failure from undermining and the subsequent loss of the supporting bank material. Although it is rigid, grouted riprap is not extremely strong: therefore, the loss of even a small area of bank support can cause failure of large portions of the revetment. See the INDOT *Standard Specifications* for more information.

38-4.06 Grouted Fabric Slope Pavement

Grouted fabric slope pavement revetments are constructed by injecting sand-cement mortar between two layers of double-woven fabric which has first been positioned on the slope to be protected. Mortar may be injected into this fabric envelope either underwater or in-the-dry. The fabric enclosure prevents dilution of the mortar during placement underwater. The two layers of fabric act first as the top and bottom form to hold the mortar in place while it hardens. This fabric, to which the mortar remains tightly bonded, then acts as tensile reinforcing to hold the mortar in place on the slope. These revetments are analogous to slope paving with reinforced concrete. The bottom layer of fabric acts as a filter cloth underlayment to prevent loss of soil particles through any cracks which may develop in the revetment as a result of soil subsidence. Often, greater relief of hydrostatic uplift is provided by weep holes or filter points which are normally woven into the fabric and remain unobstructed by mortar during the filling operation.

38-4.07 Sand-Cement Bags

Sand cement bags generally consist of a dry mix of sand and cement placed in a burlap or other suitable bag. They are handplaced in contact with adjacent bags. They require firm support from the protected bank. Usually a filter fabric is placed underneath this type of riprap. Adequate protection of the terminals and toe is essential. The riprap has little flexibility and low tensile strength and is susceptible to damage, particularly on flatter slopes where the area of contact between the bags is less.

38-4.08 Soil Cement

Soil-cement generally consists of a dry mix of sand and cement and admixtures batched in a central mixing plant. It is usually transported, placed by equipment capable of producing the width and thickness required and compacted to the required density. Control of the moisture and time after introduction of the mixing water is critical. Curing is required. This results in a rigid protection. Soil-cement can be placed either as a lining or in stepped horizontal layers. The stepped horizontal layers are extremely stable provided toe scour protection has been incorporated in the design.

38-5.0 DESIGN CONCEPTS

38-5.01 Introduction

Design concepts related to the design of bank protection are discussed in this Section. Subjects covered in this Section include design discharge, flow types, section geometry, flow in channel bends, flow resistance and extent of protection.

38-5.02 Design Discharge

Design flow rates for the design or analysis of highway structures in the vicinity of rivers and streams usually have a 10- to 100-year recurrence interval. In most cases, these discharge levels will also be applicable to the design of revetment systems. However, the designer should be aware that, in some instances, a lower discharge may produce hydraulically worse conditions with respect to riprap stability. It is suggested that several discharge levels be evaluated to ensure that the design is adequate for all discharge conditions up to that selected as the design discharge for structures associated with the riprap scheme.

38-5.03 Flow Types

Open channel flow can be classified from three points of reference. These are as follows:

1. uniform, gradually varying or rapidly varying flow;
2. steady or unsteady flow; and
3. subcritical or supercritical flow.

Design relationships presented in this Chapter are based on the assumption of uniform, steady, subcritical flow. These relationships are also valid for gradually varying flow conditions. Although the individual hydraulic relationships presented are not in themselves applicable to rapidly varying, unsteady or supercritical flow conditions, procedures are presented for extending their use to these flow conditions (see Chapter Thirty for more details related to channel design).

Rapidly varying, unsteady flow conditions are common in areas of flow expansion, flow contraction and reverse flow. These conditions are common at and immediately downstream of bridges. Supercritical or near supercritical flow conditions are common at bridge constrictions and on steep sloped channels.

Non-uniform, unsteady and near supercritical flow conditions create stresses on the channel boundary that are significantly different from those induced by uniform, steady, subcritical flow. These stresses are difficult to assess quantitatively. The stability factor method of riprap design presented in Section 38-6.0 provides a means of adjusting the final riprap design (which is based on relationships derived for steady, uniform, subcritical flow) for the uncertainties associated with these other flow conditions. The adjustment is made through the assignment of a stability factor. The magnitude of the stability factor is based on the level of uncertainty inherent in the design flow conditions.

38-5.04 Section Geometry

Design procedures presented in this Chapter require as input channel cross section geometry. The cross section geometry is necessary to establish the hydraulic design parameters (flow depth, top width, velocity, hydraulic radius, etc.) required by the riprap design procedures and to establish a construction cross section for placement of the revetment material. Where the entire channel perimeter will be stabilized, the selection of an appropriate channel geometry is only a function of the desired channel conveyance properties and any limiting geometric constraints. However, when the channel bank alone will be protected, the design must consider the existing channel bottom geometry.

The development of an appropriate channel section for analysis is very subjective. The intent is to develop a section which reasonably simulates a worst-case condition with respect to riprap stability. Information which can be used to evaluate channel geometry includes current channel surveys, past channel surveys (if available), and current and past aerial photos. In addition, the effect channel stabilization will have on the local channel section must be considered.

The first problem arises when an attempt is made to establish an existing channel bottom profile for use in design. A single channel profile is usually not enough to establish the design cross section. In addition to current channel surveys, historic surveys can provide valuable information. A comparison of current and past channel surveys at the location provides information on the general stability of the site and a history of past channel geometry changes. Often, past surveys at a particular site will not be available. If this is the case, past surveys at other sites in the vicinity of the design location may be used to evaluate past changes in channel geometry.

38-5.05 Flow In Channel Bends

Flow conditions in channel bends are complicated by the distortion of flow patterns in the vicinity of the bend. In long, relatively straight channels, the flow conditions are uniform and symmetrical about the center line of the channel. However, in channel bends, the centrifugal forces and secondary currents produced lead to non-uniform and non-symmetrical flow conditions.

Special consideration must be given to the increased velocities and shear stresses that are generated as a result of non-uniform flow in bends.
Superelevation of flow in channel bends is another important consideration in the design of revetments. Although the magnitude of superelevation is generally small when compared with the overall flow depth in the bend (usually less than 0.3 m), it should be considered when establishing freeboard limits for bank protection schemes on sharp bends. The magnitude of superelevation at a channel bend may be estimated for subcritical flow by the following equation.

$$Z = C\,[(V_a^2 T)/(gR_o)] \qquad \text{(Equation 38-5.1)}$$

Where: Z = superelevation of the water surface, m

C	=	coefficient that relates free vortex motion to velocity streamlines for unequal radius of curvature
V_a	=	mean channel velocity, m/s
T	=	water-surface width at section, m
g	=	gravitational acceleration (9.81 m/s^2)
R_o	=	the mean radius of the channel centerline at the bend, m

The coefficient C has been evaluated by the U.S. Geological Survey (USGS) and ranges between 0.5 and 3.0, with an average value of 1.5.

38-5.06 Flow Resistance

The hydraulic analysis performed as a part of the riprap design process requires the estimation of Manning's roughness coefficient. Physical characteristics upon which the resistance equations are based include the channel base material, surface irregularities, variations in section geometry, bed form, obstructions, vegetation, channel meandering, flow depth and channel slope. In addition, seasonal changes in these factors must also be considered. See Chapter Thirty for a discussion on the selection of Manning's n values.

38-5.07 Extent of Protection

Extent of protection refers to the longitudinal and vertical extent of protection required to adequately protect the channel bank.

38-5.07(01) Longitudinal Extent

The longitudinal extent of protection required for a particular bank protection scheme is highly dependent on local site conditions. In general, the revetment should be continuous for a distance greater than the length that is impacted by channel-flow forces severe enough to cause dislodging and/or transport of bank material. Although this is a vague criteria, it demands serious consideration. Review of existing bank protection sites has revealed that a common misconception in streambank protection is to provide protection too far upstream and not far enough downstream.

One criteria for establishing the longitudinal limits of protection required is illustrated in Figure 38-5A. As illustrated, the minimum distances recommended for bank protection are an upstream distance of 1.0 channel width and a downstream distance of 1.5 channel widths from corresponding reference lines. All reference lines pass through tangents to the bend at the bend entrance or exit. This criteria is based on an analysis of flow conditions in symmetric channel bends under ideal laboratory conditions. Real-world conditions are rarely as simplistic.

In actuality, many site-specific factors have a bearing on the actual length of bank that should be protected. A designer will find the above criteria difficult to apply on mildly curving bends or on channels having irregular, non-symmetric bends. Also, other channel controls (such as bridge abutments) might already be producing a stabilizing effect on the bend so that only a part of the channel bend needs to be stabilized. In addition, the magnitude or nature of the flow event might only cause erosion problems in a very localized portion of the bend, requiring that only a short channel length be stabilized. Therefore, the above criteria should only be used as a starting point. Additional analysis of site-specific factors is necessary to define the actual extent of protection required.

Field reconnaissance is a useful tool for the evaluation of the longitudinal extent of protection required, particularly if the channel is actively eroding. In straight channel reaches, scars on the channel bank may be useful to help identify the limits required for channel bank protection. In this case, it is recommended that upstream and downstream limits of the protection scheme be extended a minimum of one channel width beyond the observed erosion limits.

In curved channel reaches, the scars on the channel bank can be used to establish the upstream limit of erosion. Here a minimum of one channel width should be added to the observed upstream limit to define the limit of protection. The downstream limit of protection required in curved channel reaches is not as easy to define. Because the natural progression of bank erosion is in the downstream direction, the present visual limit of erosion might not define the ultimate downstream limit. Additional analysis based on consideration of flow patterns in the channel bend may be required.

38-5.07(02) Vertical Extent

The vertical extent of protection required of a revetment includes design height and foundation or toe depth.

1. Design Height. The design height of a riprap installation should be equal to the design highwater elevation plus some allowance for freeboard. Freeboard is provided in causeway situations to ensure that the desired degree of protection will not be reduced by unaccounted factors, including the following:

    a. wave action (from wind or boat traffic);
    b. superelevation in channel bends;

c. hydraulic jumps; and

d. flow irregularities due to piers, transitions and flow junctions.

In addition, erratic phenomena such as unforeseen embankment settlement, the accumulation of silt, trash, debris in the channel, aquatic or other growth in the channels, and ice flows should be considered when setting freeboard heights. Also, wave run-up on the bank must be considered.

The prediction of wave heights from boat-generated waves is not as straightforward as other wave sources. Figure 38-5B provides a definition sketch for the wave height discussion to follow. The height of boat-generated waves must be estimated from observations.

It is necessary to estimate the magnitude of wave runup which results when waves impact the bank. Wave runup is a function of the design wave height, the wave period, bank angle and the bank surface characteristics (as represented by different revetment materials). For wave heights less than 0.6 m, wave runup can be computed using Figure 38-5C and Figure 38-5D. The runup height (R) given in Figure 38-5D is for concrete pavement. Correction factors are provided in Figure 38-5C for reducing the runup magnitude for other revetment materials. The correction factor is multiplied times the wave height to get the resulting wave runup (R).

As indicated, there are many factors which must be considered in the selection of an appropriate freeboard height. As a minimum, it is recommended that a freeboard elevation of 0.3 m to 0.6 m be used in unconstricted reaches, and 0.6 m to 0.9 m in constricted reaches. The Federal Emergency Management Agency requires 0.9 m for levee protection and 1.2 m at bridges for the 100-year flood. When computational procedures indicate that additional freeboard may be required, the greater height should be used. In addition, it is recommended that the designer observe wave and flow conditions during various seasons of the year (if possible), consult existing records, and interview persons who have knowledge of past conditions when establishing the necessary vertical extent of protection required for a particular revetment installation.

2. Toe Depth. The undermining of revetment toe protection has been identified as one of the primary mechanisms of revetment failure. In the design of bank protection, estimates of the depth of scour are needed so that the protective layer is placed sufficiently low in the streambed to prevent undermining. The ultimate depth of scour must consider channel degradation and natural scour and fill processes.

The relationships presented in Equations 38-5.2 and 38-5.3 can be used to estimate the probable maximum depth of scour due to natural scour and fill phenomenon in straight channels and in channels having mild bends. In application, the depth of scour, d_s, should be measured from the lowest elevation in the cross section. It should be assumed that the low point in the cross section may eventually move adjacent to the protection (even if this is not the case in the current survey).

$$d_s = 3.66 \text{ m} \qquad \text{for } D_{50} < 1.5 \text{ mm} \qquad \text{(Equation 38-5.2)}$$

$$d_s = 3.72\, D_{50}^{-0.11} \qquad \text{for } D_{50} > 1.5 \text{ mm} \qquad \text{(Equation 38-5.3)}$$

Where:
d_s = estimated probable maximum depth of scour, m
D_{50} = median diameter of bed material, mm

(Note: If D_{50} is in meters, then $d_s = 1.738\, D_{50}^{-0.11}$.)

The depth of scour predicted by Equations 38-5.2 and 38-5.3 must be added to the magnitude of predicted degradation and local scour (if any) to arrive at the total required toe depth.

38-6.0 DESIGN GUIDELINES

38-6.01 Rock Riprap

This Section contains design guidelines for the design of rock riprap. Guidelines are provided for bank slope, rock size, rock gradation, riprap layer thickness, filter design, edge treatment and construction considerations. In addition, typical construction details are illustrated. In most cases, the guidelines presented apply equally to rock and rubble riprap.

38-6.01(01) Bank Slope

A primary consideration in the design of stable riprap bank protection schemes is the slope of the channel bank. For riprap installations, normally the maximum recommended face slope is 2H:1V. Although generally not recommended, the steepest slope acceptable for rubble revetment is 1.5H:1V. To be stable under an identical wave attack or lateral velocity, a rubble revetment with a steep slope will need larger rubble sizes and greater thicknesses than one with a flatter slope.

38-6.01(02) Rock Size

The stability of a particular riprap particle is a function of its size, expressed either in terms of its weight or equivalent diameter. Relationships are presented for evaluating the riprap size required to resist particle and wave erosion forces.

1. Particle Erosion. Two methods or approaches have been used historically to evaluate a material's resistance to particle erosion - the permissible velocity approach and the permissible tractive force (shear stress) approach. Under the permissible velocity approach the channel is assumed stable if the computed mean velocity is lower than the maximum permissible velocity. The tractive force (boundary shear stress) approach focuses on stresses developed at the interface between flowing water and materials forming the channel boundary.

2. Design Relationship. A riprap design relationship that is based on tractive force theory yet has velocity as its primary design parameter is presented in Equation 38-6.1. The design relationship in Equation 38-6.1 is based on the assumption of uniform, gradually varying flow. Figure 38-6A presents a graphical solution to Equation 38-6.1. Equation 38-6.2 can be solved using Figures 38-6B and 38-6C.

$$D_{50} = 0.00594\ V_a^3 / (d_{avg}^{0.5}\ K_1^{1.5}) \qquad \text{(Equation 38-6.1)}$$

Where:
- D_{50} = the median riprap particle size, m
- C = correction factor (described below)
- V_a = the average velocity in the main channel, m/s
- d_{avg} = the average flow depth in the main flow channel, m

K_1 is defined as:

$$K_1 = [1 - (\sin^2 \theta / \sin^2 \varphi)]^{0.5} \qquad \text{(Equation 38-6.2)}$$

Where:
- θ = the bank angle with the horizontal
- φ = the riprap material's angle of repose

The average flow depth and velocity used in Equation 38-6.1 are main channel values. The main channel is defined as the area between the channel banks (see Figure 38-6D).

Equation 38-6.1 is based on a rock riprap specific gravity of 2.65 and a stability factor of 1.2. Equations 38-6.3 and 38-6.4 present correction factors for other specific gravities and stability factors:

$$C_{sg} = 2.12 / (S_s - 1)^{1.5} \qquad \text{(Equation 38-6.3)}$$

Where: S_s = the specific gravity of the rock riprap

$$C_{sf} = (SF / 1.2)^{1.5} \qquad \text{(Equation 38-6.4)}$$

Where: SF = the stability factor to be applied.

The correction factors computed using Equations 38-6.3 and 38-6.4 are multiplied together to form a single correction factor C. This correction factor, C, is then multiplied by the riprap size computed from Equation 38-6.1 to arrive at a stable riprap size. Figure 38-6E provides a solution to Equations 38-6.3 and 38-6.4 using correction factor C.

The stability factor, SF, used in Equations 38-6.1 and 38-6.4 requires additional explanation. The stability factor is defined as the ratio of the average tractive force exerted by the flow field and the riprap material's critical shear stress. As long as the stability factor is greater than 1, the critical shear stress of the material is greater than the flow-induced tractive stress, and the riprap is considered to be stable. As mentioned above, a stability factor of 1.2 was used in the development of Equation 38-6.1.

The stability factor is used to reflect the level of uncertainty in the hydraulic conditions at a particular site. Equation 38-6.1 is based on the assumption of uniform or gradually varying flow. In many instances, this assumption is violated or other uncertainties come to bear. For example, debris and/or ice impacts, or the cumulative effect of high shear stresses and forces from boat-generated waves. The stability factor is used to increase the design rock size when these conditions must be considered. Figure 38-6F presents guidelines for the selection of an appropriate value for the stability factor.

3. Application. Application of the relationship in Equation 38-6.1 is limited to uniform or gradually varying flow conditions that are in straight or mildly curving channel reaches of relatively uniform cross section. However, design needs dictate that the relationship also be applicable in nonuniform, rapidly varying flow conditions often exhibited in natural channels with sharp bends and steep slopes and in the vicinity of bridge piers and abutments.

   To fill the need for a design relationship that can be applied at sharp bends and on steep slopes in natural channels and at bridge abutments, it is recommended that Equation 38-6.1 be used with appropriate adjustments in velocity and/or stability factor as outlined in the following sections.

4. Steep Slopes. Flow conditions in steep sloped channels are rarely uniform, and they are characterized by high flow velocities and significant flow turbulence. In applying Equation 38-6.1 to steep sloped channels, care must be exercised in the determination of an appropriate velocity. When determining the flow velocity in steep sloped channels, it is

recommended that Equation 38-6.5 be used to determine the channel roughness coefficient. It is also important to thoughtfully consider the guidelines for selection of stability factors as presented in Figure 38-6F.

On high gradient streams it is extremely difficult to obtain a good estimate of the median bed material size. For high gradient streams with slopes greater then 0.002 and bed material larger than 0.06 m (gravel, cobble or boulder size material), it is recommended that the relationship given in the following equation be used to evaluate the base Manning's n.

$$n = 0.32\ S_f^{0.38}\ R^{-0.16} \qquad \text{(Equation 38-6.5)}$$

Where:
- S_f = friction slope, m/m
- R = hydraulic radius, m

5. Bridge Piers. For recommendations, see Chapter Thirty-two.

6. Wave Erosion. Waves generated by boat traffic have also been observed to cause bank erosion on inland waterways. The most widely used measure of riprap's resistance to waves is that developed by R. Y. Hudson "Laboratory Investigations of Rubble-Mound Breakwaters," 1959. The so-called Hudson relationship is given by the following equation.

$$W_{50} = (\gamma_s\ H^3) / (2.20\ [S_s - 1]^3 \cot \theta) \qquad \text{(Equation 38-6.6)}$$

Where:
- W_{50} = weight of the median particle, kg
- γ_s = unit weight of riprap (solid) material, kg/m^3 (other parameters are as defined previously)
- H = the wave height, m
- S_s = specific gravity of riprap material
- θ = bank angle with the horizontal

Assuming: $S_s = 2.65$ and $\gamma_s = 2643$ kg/m^3, Equation 38-6.6 can be reduced to:

$$W_{50} = 267.4\ H^3/\cot\theta \qquad \text{(Equation 38-6.7)}$$

In terms of an equivalent diameter, Equation 38-6.7 can be reduced to the following:

$$D_{50} = 0.75H/\cot^{1/3}\theta \qquad \text{(Equation 38-6.8)}$$

Where: D_{50} = median riprap size, m

Methods for estimating a design wave height are presented in Section 38-5.07. Equation 38-6.8 is presented in nomograph form in Figure 38-6G. Equations 38-6.7 and 38-6.8 can be used for preliminary or final design when H is less than 1.5 m, and there is no major overtopping of the embankment.

7. Ice Damage. Ice can affect riprap linings in a number of ways. Moving surface ice can cause crushing and bending forces and large impact loadings. The tangential flow of ice along a riprap-lined channel bank can also cause excessive shearing forces. Quantitative criteria for evaluating the impact ice has on channel protection schemes are unavailable. However, historic observations of ice flows in New England rivers indicate that riprap sized to resist design flow events will also resist ice forces.

   For design, consideration of ice forces should be evaluated on a case-by-case basis. In most instances, ice flows are not of sufficient magnitude to warrant detailed analysis. Where ice flows have historically caused problems, a stability factor of 1.2 to 1.5 should be used to increase the design rock size. Please note that the selection of an appropriate stability factor to account for ice-generated erosive problems should be based on local experience.

38-6.01(03) Rock Gradation

The gradation of stones in riprap revetment affects the riprap's resistance to erosion. The stone should be reasonably well graded throughout the riprap layer thickness. The gradation limits should not be so restrictive that production costs would be excessive. Figure 38-6H presents suggested guidelines for establishing gradation limits. Figure 38-6I presents three suggested gradation classes based on INDOT specifications. Figure 38-6J can be used as an aid in selecting appropriate gradation limits.

It is recognized that the use of a four-point gradation as specified in Figure 38-6H might, in some cases, be too harsh a specification for some smaller quarries. If this is the case, the 85 percent specification can be dropped as is done in Figure 38-6I. In most instances, a uniform gradation between D_{50} and D_{100} will result in an appropriate D_{85}.

38-6.01(04) Layer Thickness

All stones should be contained reasonably well within the riprap layer thickness to provide maximum resistance against erosion. Oversized stones, even in isolated spots, may cause riprap failure by precluding mutual support between individual stones, providing large voids that expose filter and bedding materials, and creating excessive local turbulence that removes smaller stones. Small amounts of oversized stone should be removed individually and replaced with proper size stones. The following criteria apply to the riprap layer thickness.

1. It should not be less than the spherical diameter of the D_{100} (W_{100}) stone or less than 2.0 times the spherical diameter of the D_{50} (W_{50}) stone, whichever results in the greater thickness.

2. It should not be less than 300 mm for practical placement.

3. The thickness determined by either Item 1 or 2 should be increased by 50 percent when the riprap is placed underwater to provide for uncertainties associated with this type of placement.

4. An increase in thickness of 150 mm to 300 mm, accompanied by an appropriate increase in stone sizes, should be provided where riprap revetment will be subject to attack by floating debris or ice or by waves from boat wakes, wind or bedforms.

38-6.01(05) Filter Design

A filter is a transitional layer of gravel, small stone or fabric placed between the underlying soil and the structure. The filter prevents the migration of the fine soil particles through voids in the structure, distributes the weight of the armor units to provide more uniform settlement, and permits relief of hydrostatic pressures within the soils. A filter should be used wherever the riprap is placed on noncohesive material subject to significant subsurface drainage (such as in areas where water surface levels fluctuate frequently and in areas of high groundwater levels).

1. Granular Filters. For rock riprap, a filter ratio of 5 or less between layers will usually result in a stable condition. The filter ratio is defined as the ratio of the 15 percent particle size (D_{15}) of the coarser layer to the 85 percent particle size (D_{85}) of the finer layer. An additional requirement for stability is that the ratio of the 15 percent particle size of the coarser material to the 15 percent particle size of the finer material should exceed 5 but be less than 40. These requirements can be stated as follows:

$$\frac{D_{15}\,(\text{coarser layer})}{D_{85}\,(\text{finer layer})} < 5 < \frac{D_{15}\,(\text{coarser layer})}{D_{15}\,(\text{finer layer})} < 40 \qquad \text{(Equation 38-6.9)}$$

The first test of the inequality is intended to prevent piping through the filter; the right portion of the second test provides for adequate permeability for structural bedding layers; and the right portion provides a uniformity criteria.

If a single layer of filter material will not satisfy the filter requirements, one or more additional layers of filter material must be used. The filter requirement applies between the bank material and the filter blanket, between successive layers of filter material if more than one layer is used, and between the filter blanket and the riprap cover. In addition to the filter requirements, the grain size curves for the various layers should be approximately parallel to minimize the infiltration of fine material from the finer layer to the coarser layer. Not more than 5 percent of the filter material should pass the 75-μm sieve. Figures 38-6J and 38-6K can be used as an aid in designing an appropriate granular filter.

The thickness of the filter blanket should range from 150 mm to 450 mm for a single layer or from 100 mm to 200 mm for individual layers of a multiple layer blanket. Where the gradation curves of adjacent layers are approximately parallel, the thickness of the blanket layers should approach the minimum. The thickness of individual layers should be increased above the minimum proportionately as the gradation curve of the material comprising the layer departs from a parallel pattern.

2. Geotextile Filters Synthetic geotextile filters have found considerable use as alternatives to granular filters. See Figure 38-6L. Since the original geotextile erosion control application in 1957, thousands of successful projects have been completed. The following advantages relevant to using geotextile filters have been identified.

   a. Installation is generally quick and labor efficient.

   b. Geotextile filters are more economical than granular filters.

   c. Geotextile filters have consistent and more reliable material quality.

   d. Geotextile filters have good inherent tensile strength.

   e. Local availability of suitable granular filter material is no longer a design consideration when using fabric filters.

   Disadvantages include the following:

   a. Geotextiles can be difficult to install underwater.

b. Geotextiles have widely variable hydraulic properties and must be designed based on project- specific conditions and performance requirements.

c. Geotextile filter performance is sensitive to construction procedures.

d. Special installation and inspection procedures may be necessary when using geotextile filters.

3. Geotextile Filter Design The design of geotextile filters closely follows traditional graded granular filter design principles and should consider the following performance areas.

a. soil retention (piping resistance),
b. permeability,
c. clogging, and
d. survivability.

It is extremely desirable that individual site conditions and performance requirements be established in conjunction with the geotextile design. Generalized geotextile requirements should be used only on very small or non-critical/non-severe installations where a detailed analysis is not warranted. AASHTO has developed materials and construction specifications (AASHTO Specification M 288) for routine, non-critical/non-severe geotextile applications. Details of geotextile filter design, for all levels of project severity and criticality, are presented in the Federal Highway Administration publication *Geosynthetic Design and Construction Guidelines*, (FHWA-HI-95-038). This reference provides detailed guidance on specifying and installing geotextiles for a variety of transportation applications. The American Society for Testing and Materials Committee D-35 has developed standard testing procedures for approximately 35 general, index and performance properties of geosynthetics. These standard test procedures are recommended for use in design and specifications when using geosynthetics.

The following design steps are necessary for geotextile design in riprap and other permanent erosion control applications.

Step 1: Evaluate the application site (determine if the application is critical or severe).

Step 2: Obtain and test soil samples (perform grain size analysis and permeability tests).

Step 3: Evaluate possible armor material and placement procedures.

Step 4: Calculate anticipated reverse flow through the erosion control system.

Step 5: Determine geotextile requirements as follows:

(1) soil retention,
(2) permeability/permittivity,
(3) clogging, and
(4) survivability.

Step 6: Estimate cost and prepare specifications.

4. Geotextile Installation Procedures. To provide good performance, a properly selected cloth should be installed with due regard for the following precautions.

   a. Grade area and remove debris to provide a smooth, fairly even surface.

   b. Place geotextile loosely, laid with the machine (generally roll) direction in the direction of anticipated water flow or movement.

   c. Seam or overlap the geotextile as required.

   d. The maximum allowable slope on which a riprap-geotextile system can be placed is equal to the lowest soil-geotextile friction angle for the natural ground or stone-geotextile friction angle for cover (armor) materials. Additional reductions in slope may be necessary due to hydraulic considerations and possible long-term stability. For slopes greater than 2.5H:1V, special construction procedures will be required.

   e. For streambank and wave action applications, the geotextile must be keyed in at the bottom of the slope. If the system can not be extended a few meters above anticipated high-water level, the geotextile should also be keyed in at the crest of the slope.

   f. Place the revetment (cushion layer and/or riprap) over the geotextile width while avoiding puncturing it.

38-6.01(06) Edge Treatment

The edges of riprap revetments (flanks, toe and head) require special treatment to prevent undermining. The flanks of the revetment should be designed as illustrated in Figure 38-6M. The upstream flank is illustrated in section (a) and the downstream flank in section (b) of this figure. A more constructible flank section uses riprap rather than compacted fill.

Undermining of the revetment toe is one of the primary mechanisms of riprap failure. The toe of the riprap should be designed as illustrated in Figure 38-6N. The toe material should be placed in a toe trench along the entire length of the riprap blanket.

Where a toe trench cannot be dug, the riprap blanket should terminate in a thick, stone toe at the level of the streambed (see alternate design in Figure 38-6N). Care must be taken during the placement of the stone to ensure that the toe material does not mound and form a low dike; a low dike along the toe could result in flow concentration along the revetment face which could stress the revetment to failure. In addition, care must be exercised to ensure that the channel's design capability is not impaired by placement of too much riprap in a toe mound.

The size of the toe trench or the alternate stone toe is controlled by the anticipated depth of scour along the revetment. As scour occurs (and in most cases it will), the stone in the toe will launch into the eroded area as illustrated in Figure 38-6 O. Observation of the performance of these types of rock toe designs indicates that the riprap will launch to a final slope of approximately 2H:1V.

The volume of rock required for the toe must be equal to or exceed one and one-half times the volume of rock required to extend the riprap blanket (at its design thickness and on a slope of 2H:1V) to the anticipated depth of scour. Dimensions should be based on the required volume using the thickness and depth determined by the scour evaluation. The alternate location can be used when the amount of rock required would not constrain the channel. Establishing a design scour depth is covered in Section 38-5.07.

38-6.01(07) Construction Considerations

Construction considerations related to the construction of riprap revetments include bank slope or angle, bank preparation and riprap placement.

1. Bank Preparation. The bank should be prepared by first clearing all trees and debris from the bank and grading the bank surface to the desired slope. In general, the graded surface should not deviate from the specified slope line by more than 150 mm. However, local depressions larger than this can be accommodated because initial placement of filter material and/or rock for the revetment will fill these depressions. In addition, any large boulders or debris found buried near the edges of the revetment should be removed.

2. Riprap Placement. The common methods of riprap placement are hand placing; machine placing, such as from a skip, dragline or some form of bucket; and dumping from trucks and spreading by bulldozer. Hand placement produces the best riprap revetment, but it is the most expensive method except when labor is unusually cheap. Steeper side slopes can

be used with hand-placed riprap than with other placing methods. Where steep slopes are unavoidable (when channel widths are constricted by existing bridge openings or other structures and when rights-of-way are costly), hand placement should be considered.

In the machine placement method, sufficiently small increments of stone should be released as close to their final positions as practical. Rehandling or dragging operations to smooth the revetment surface tend to result in segregation and breakage of stone and can result in a rough revetment surface. Stone should not be dropped from an excessive height because this may result in the same undesirable conditions. Riprap placement by dumping with spreading is satisfactory provided the required layer thickness is achieved. Riprap placement by dumping and spreading is the least desirable method because a large amount of segregation and breakage can occur and is not recommended. In some cases, it may be economical to increase the layer thickness and stone size somewhat to offset the shortcomings of this placement method.

38-6.01(08) Design Procedure

The rock riprap design procedure outlined in the following sections is comprised of three primary sections - preliminary data analysis, rock sizing and revetment detail design. The individual steps in the procedure are numbered consecutively throughout each of the sections. Figures 38-6P and 38-6Q provide a useful format for recording data at each step of the analysis.

1. Compile all necessary field data including channel cross section surveys, soils data, aerial photographs, history of problems at site, etc.

2. Determine design discharge.

3. Develop design cross section(s). Note: The rock sizing procedures described in the following steps are designed to prevent riprap failure from particle erosion.

4. Compute design water surface as follows:

    a. When evaluating the design water surface, Manning's "n" should be estimated. If a riprap lining is being designed for the entire channel perimeter, an estimate of the rock size may be required to determine the roughness coefficient "n" (Section 38-5.06).

    b. If the design section is a regular trapezoidal shape, and flow can be assumed to be uniform, use design procedures from Chapter Thirty.

c. If the design section is irregular or flow is not uniform, backwater procedures must be used to determine the design water surface.

d. Any backwater analysis conducted must be based on conveyance weighing of flows in the main channel, right bank and left bank.

5. Determine design average velocity and depth as follows:

   a. Average velocity and depth should be determined for the design section in conjunction with the computations of Step 4. In general, the average depth and velocity in the main flow channel should be used.

   b. If riprap is being designed to protect channel banks, abutments or piers located in the floodplain, average floodplain depths and velocities should be used.

6. Compute the bank angle correction factor K_1 (Equation 38-6.2 and Figures 38-6B and 38-6C).

7. Determine riprap size required to resist particle erosion (Equation 38-6.1, Figure 38-6A) as follows:

   a. Initially assume no corrections.

   b. Evaluate correction factor for rock riprap specific gravity and stability factor ($C = C_{sg}C_{sf}$).

   c. If designing riprap for piers or abutments, see Chapter Thirty-two.

8. If entire channel perimeter is being stabilized, and an assumed D_{50} was used in determination of Manning's "n" for backwater computations, return to Step 4 and repeat Steps 4 through 7.

9. If causeway situation, determine wave height.

10. Select final D_{50} riprap size, set material gradation (see Section 38-6.01(03) and Figure 38-6J), and determine riprap layer thickness (see Section 38-6.01(05)).

11. Determine longitudinal extent of protection required (Section 38-5.07).

12. Determine appropriate vertical extent of revetment (Section 38-5.07).

13. Design filter layer (Section 38-6.01(05), Figure 38-6K):

a. Determine appropriate filter material size and gradation.
b. Determine layer thickness.

14. Design edge details (flanks and toe) (Section 38-6.01(07)).

38-6.01(09) Design Examples

The following design examples illustrate the use of the design methods and procedures outlined above. Two examples are given. Example 1 illustrates the design of a riprap-lined channel section. Example 2 illustrates the design of riprap as bank protection. In the examples, the steps correlate with the design procedure presented above. Computations are also shown on appropriate figures.

1. Example 1. A 380-m channel reach is to be realigned to make room for the widening of an existing highway. Realignment of the channel reach will necessitate straightening the channel and reducing its length from 380 m to 305 m. The channel is to be sized to carry 140 m^3/s within its banks. Additional site conditions are as follows:

   a. flow conditions can be assumed to be uniform or gradually varying;

   b. the existing channel profile dictates that the straightened reach be designed at a uniform slope of 0.0049;

   c. the natural soils are gap graded from medium sands to coarse gravels giving the following distribution:

   D_{85} = 32 mm D_{50} = 19.5 mm D_{15} = 1.37 mm
   K (permeability) = 3.5×10^{-2} cm/s

   d. Available rock riprap has a specific gravity of 2.65 and D_{50} = 0.3 m.

   Design a stable trapezoidal riprap-lined channel for this site. Design figures used to summarize data in this example are reproduced in Figures 38-6R and 38-6S.

   a. Compile Field Data.

      (1) See given information for this example.

(2) Other field data would typically include site history, geometric constraints, roadway crossing profiles, site topography, etc.

b. Design Discharge.

(1) Given as 140 m^3/s.

(2) Discharge in main channel equals the design discharge because entire design discharge is to be contained in channel as specified.

c. Design Cross Section.

(1) As specified, a trapezoidal section is to be designed.

(2) Initially assume a trapezoidal section with 6.1 m bottom width and 2H:1V side slopes (see Figure 38-6R).

d. Compute Design Water Surface.

(1) Determine roughness coefficient (n = 0.04).

(2) Compute flow depth. Assume R = 2.1 m

(3) Solve Manning's equation for normal depth or see Chapter Thirty:

$Q = (1/n)\, A\, R^{2/3}\, S^{1/2}$
$d = 3.61$ m

(4) Compute hydraulic radius to compare with the assumed value used in Step d(1) (use computer programs, available charts and tables, or manually compute).

$R = A/P$
$R = 47.8 / 22.1$
R = 2.16 m, which is approximately equal to R (assumed); therefore,
d = 3.61 m OK

e. Determine Design Parameters.

$A = 3.61(6.1) + 2\,(3.61)^2 = 48.1\ m^2$
$V_a = Q/A = 140 / 48.1 = 2.91$ m/s
$d_a = d = 3.61$ m (uniform channel bottom)

f. Bank Angle Correction Factor.

θ = 2H:1V
φ = 41^{o} (from Figure 38-6B)
K_1 = 0.73 (from Figure 38-6A)

g. Determine riprap size(see Section 38-6.01(08)).

(1) Using Figure 38-6S:

for channel bed, D_{50} = 85 mm
for channel bank, D_{50} = 131 mm

(2) Riprap specific gravity = 2.65 (given):

Stability factor = 1.2 (Column 9, Figure 38-6P)
(uniform flow, little or no uncertainty in design)
C = 1

(3) No piers or abutments to evaluate for this example; therefore:

$C_{p/a}$ = 1

(4) Corrected riprap size:

For channel bed, $D'_{50} = D_{50}$ = 85 mm
For channel banks, $D'_{50} = D_{50}$ = 131 mm

h. This step is not applicable.

i. Surface waves. Surface waves determined not to be a problem at this site.

j. Select Design Riprap Size, Gradation and Layer Thickness.

D_{50} size: Recommend AASHTO Face Class riprap
D_{50} = 290 mm (for entire perimeter)

Layer thickness (T):

$T = 2D_{50}$ = 2(290 mm) = 580 mm, or $T = D_{100}$ = 395 mm
Use T = 600 mm

k. Longitudinal Extent of Protection. Riprap lining to extend along entire length of straightened reach plus some additional upstream and downstream distance.

l. Vertical Extent of Protection. Riprap entire channel perimeter to top-of-bank.

m. Filter Layer Design.

(1) Filter material size.

$$\frac{D_{15}\,(coarser\ layer)}{D_{85}\,(finer\ layer)} < 5 < \frac{D_{15}\,(coarser\ layer)}{D_{15}\,(finer\ layer)} < 40 \qquad \text{(Equation 38-6.10)}$$

(2) For riprap to soil interface:

$$\frac{D_{15}\,(riprap)}{D_{85}\,(soil)} = \frac{183}{30} = 6 > 5 \qquad \text{(Equation 38-6.11)}$$

and

$$\frac{D_{15}\,(riprap)}{D_{15}\,(soil)} = \frac{183}{1.37} = 133 > 40 \qquad \text{(Equation 38-6.12)}$$

Therefore, a filter layer is needed. Try 51 mm uniformly graded coarse gravel filter.

(3) For the filter to soil interface:

$$\frac{D_{15}\,(filter)}{D_{85}\,(soil)} = \frac{30}{32} = 0.94 < 5 \qquad \text{(Equation 38-6.13)}$$

and

$$\frac{D_{15}\,(filter)}{D_{15}\,(soil)} = \frac{30}{1.37} = 21.9 > 5\ and < 40 \qquad \text{(Equation 38-6.14)}$$

Therefore, filter to soil interface is OK.

(4) For the riprap to filter interface:

$$\frac{D_{15}(riprap)}{D_{85}(filter)} = \frac{183}{60} = 3 < 5 \qquad \text{(Equation 38-6.15)}$$

and

$$\frac{D_{15}(riprap)}{D_{15}(filter)} = \frac{183}{30} = 6 > 5 \text{ and } < 40 \qquad \text{(Equation 38-6.16)}$$

Therefore, the 51-mm filter material is adequate.

(5) Filter layer thickness. Because soil gradation curve and filter layer gradation curve are not approximately parallel, use layer thickness of 203 mm.

n. Edge Details. Line entire perimeter; edge details as per Figure 38-6M (also see sketch on Figure 38-6R).

2. Example 2. The site illustrated in Figure 38-6T and discussed below is migrating laterally towards Route 1 (see Figure 38-6T(a)). Design a riprap revetment to stabilize the active bank erosion at this site.

The process of developing an appropriate channel geometry is illustrated in Figure 38-6T. Figure 38-6T(a) illustrates the location of the design site at position "2" along Route 1. The section illustrated in Figure 38-6T(c) was surveyed at this location and represents the current condition. No previous channel surveys were available at this site. However, data from several old surveys were available in the vicinity of a railroad crossing upstream (location 1). Figure 38-6T(b) illustrates these survey data. The surveys indicate that there is a trend for the thalweg of the channel to migrate within the right half of the channel. Because locations 1 and 2 are along bends of similar radii, it can be reasonably assumed that a similar phenomenon occurs at location 2. A thalweg located immediately adjacent to the channel bank reasonably represents the worst case hydraulically for the section at location 2. Therefore, the surveyed section at location 2 is modified to reflect this. In addition, the maximum section depth (located in the thalweg) is increased to reflect the effect of stabilizing the bank. The maximum depth in the thalweg is set to 1.7 times the average depth of the original section (note that it is assumed that the average depth before modification of the section is the same as the average depth after modification). The final modified section geometry is illustrated in Figure 38-6T(c).

Additional site conditions are as follows:

a. flow conditions are gradually varying;

b. channel characteristics are as described above;

c. topographic survey indicates:

(1) channel slope = 0.0024 m/m
(2) channel width = 90 m
(3) bend radius = 365 m;

d. channel bottom is armored with cobble size material having a D_{50} of approximately 150 mm;

e. bank soils are silty sand with the following soil characteristics:

D_{85} = 1.28 mm
D_{50} = 0.46 mm
D_{15} = 0.14 mm
K (permeability) = 1.0×10^{-6} m/s;

f. available rock riprap has a specific gravity of 2.60 and is described as angular;

g. field observations indicate that the banks are severely cut just downstream of the bend apex; erosion was also observed downstream to the bend exit and upstream to the bend quarter points; and

h. bank height along cut banks is approximately 2.7 m.

Figure 38-6U presents the completed Example 2. Figures 38-6V and 38-6W summarize data used in Example 2.

a. Compile Field Data.

(1) See given information for this example.
(2) See site history given above.

b. Design Discharge.

(1) Given as 1320 m^3/s.

(2) From backwater analysis of this reach, it is determined that the discharge confined to the main channel (Q_{mc}) is 980 m^3/s.

c. Design Cross Section.

(1) Only the channel bank is to be stabilized; therefore, the channel section will consist of the existing channel with the bank graded to an appropriate angle to support the riprap revetment. Figure 38-6V illustrates the existing channel section.

(2) To minimize loss of bank vegetation and limit the encroachment of the channel on adjacent lands, a 2H:1V bank slope is to be used.

(3) As given, the current bank height along the cut banks is 2.7 m.

d. Compute Design Water Surface.

(1) Determine roughness coefficient (n = 0.042). This represents the average reach "n" used in the backwater analysis.

(2) Compute flow depth:

(a) Flow depth determined from backwater analysis. The maximum main channel depth was determined to be d_{max} = 4.57 m

(b) Hydraulic radius for main channel. R = 3.17 m (from backwater analysis). R assumed (3.05 m) is approximately equal to R actual; therefore, "n" as computed is OK.

e. Determine Other Design Parameters. From backwater analysis (all main channel values):

$A = 255\ m^2$
$V_a = 3.84$ m/s
$d_a = d = 3.66$ m

f. Bank Angle Correction Factor.

θ = 2H:1V
$\varphi = 41^o$ (from Figure 38-6B, Figure 38-6W)
$K_1 = 0.73$ (from Figure 38-6A)

g. Determine riprap size.

(1) Using Figure 38-6A, D_{50} = 274 mm

(2) Riprap specific gravity = 2.60 (given)
Stability factor = 1.6
(gradually varying flow, sharp bend - bend radius to width = 4)
C = 1.6

(3) no piers or abutments to evaluate for this example; therefore:
$C_{p/a} = 1$

(4) Corrected riprap size:

$D'_{50} = D_{50}(1.6)(1.0) = 438$

h. This step is not applicable.

i. Surface Waves. Surface waves determined not to be a problem at this site.

j. Select Design Riprap Size, Gradation and Layer Thickness (Preliminary Design of Waterway Area).

D_{50} size: Recommend AASHTO 0.23 metric ton class riprap
D_{50} = 549 mm

Gradation: See Figure 38-6U.

Layer thickness (T):
T = 2 D_{50} = 2(549) = 1098 mm or T = D_{100} = 686 mm
Use T = 1100 mm

k. Longitudinal Extent of Protection. Field observations indicate that the banks are severely cut just downstream of the bend apex; erosion was also observed downstream to the bend exit and upstream to the bend quarter points. Therefore, establish longitudinal limits of protection to extend to a point 90 m (W) upstream of the bank entrance and to a point 135 m (1.5 W) downstream of the bend exit.

l. Vertical Extent of Protection. Riprap entire channel bank from top-of-bank to below depth of anticipated scour. Scour depth evaluated as illustrated in Section 38-5.07:

$d_s = 3.72\ D_{50}^{-0.11}$ (Equation 38-5.3)

$d_s = 3.72\ (150)^{-0.11} = 2.14$ m

Adding this to the observed maximum depth yields a potential maximum scour depth of:

4.57 + 2.14 = 6.71 m

The bank material should be run to this depth, or a sufficient volume of stone should be placed at the bank toe to protect against the necessary depth of scour.

m. Filter Layer Design.

(1) Filter material size (Figure 38-6K):

$$\frac{D_{15}(\text{coarser layer})}{D_{15}(\text{finer layer})} < 5 < \frac{D_{15}(\text{coarser layer})}{D_{15}(\text{finer layer})} < 40 \qquad \text{(Equation 38-6.17)}$$

(2) For the riprap to soil interface:

$$\frac{D_{15}(\text{riprap})}{D_{85}(\text{soil})} = \frac{150}{1.3} = 115 > 5 \qquad \text{(Equation 38-6.18)}$$

and

$$\frac{D_{15}(\text{riprap})}{D_{15}(\text{soil})} = \frac{150}{0.14} = 1094 > 40 \qquad \text{(Equation 38-6.19)}$$

Therefore, a filter layer is needed. Try 13-mm uniformly graded, fine gravel filter (gradation characteristics as illustrated in Figure 38-6J).

(3) For the filter to soil interface:

$$\frac{D_{15}(\text{filter})}{D_{85}(\text{soil})} = \frac{4.6}{1.3} = 3.5 < 5 \qquad \text{(Equation 38-6.20)}$$

and

$$\frac{D_{15}(\text{filter})}{D_{15}(\text{soil})} = \frac{4.6}{0.14} = 33 > 5 \text{ and } < 40 \qquad \text{(Equation 38-6.21)}$$

Therefore, filter to soil interface is OK.

(4) For the riprap to filter interface:

$$\frac{D_{15}(riprap)}{D_{85}(filter)} = \frac{150}{30} = 5 \leq 5 \text{ and} \qquad \text{(Equation 38-6.22)}$$

and

$$\frac{D_{15}(riprap)}{D_{15}(filter)} = \frac{150}{4.57} = 32.8 > 5 \text{ and } < 40 \qquad \text{(Equation 38-6.23)}$$

Therefore, the 13-mm filter material is adequate.

(5) Filter Layer Thickness: Because soil gradation curve and filter layer, riprap and bank soil are approximately parallel, use layer thickness of 203 mm.

n. Edge Details.

(1) Flank Details: See Figure 38-6W
(2) Toe Details: See Figure 38-6W

Anticipated scour depth below existing channel bottom at the bank (d'_s) is the depth of scour (computed in Step 12) minus the current bed elevation at the bank (see Figure 38-6V): 6.7 m - 3.7 m = 3 m

Rock quantity required below the existing bed:

$R_q = d'_s(\sin^{-1} \theta)(T)(1.5)$ (Equation 38-6.24)

Where:

R_q	=	required riprap quantity per meter of bank, m^2
θ	=	the bank angle with the horizontal, degrees
T	=	the riprap layer thickness, m

$R_q = (3)\ (2.24)\ (1.1)\ (1.5) = 11.1\ m^2$

A 1.8-m deep trapezoidal toe trench, with side slopes of 2H:1V and 1H:1V and a bottom width of 1.8 m, contains the necessary volume. Figure 38-6W illustrates the resulting toe trench detail.

38-6.02 Wire-Enclosed Rock

As described in Section 38-4.03, wire-enclosed rock (gabion) revetments consist of rectangular wire mesh baskets filled with rock. The most common types of wire-enclosed revetments are mattresses and stacked blocks. The wire cages which make up the mattresses and gabions are available from commercial manufacturers.
Rock and wire mattress revetments consist of flat wire baskets or units filled with rock that are laid end to end and side to side on a prepared channel bed and/or bank. The individual mattress units are wired together to form a continuous revetment mattress.

Stacked block gabion revetments consist of rectangular wire baskets which are filled with stone and stacked in a stepped-back fashion to form the revetment surface. They are also commonly used at the toe of embankment slopes as toe walls which help to support other upper bank revetments and prevent undermining.

38-6.02(01) Design Guidelines For Mattresses

Components of a rock and wire mattress design include layout of a general scheme or concept, bank and foundation preparation, mattress size and configuration, stone size, stone quality, basket or rock enclosure fabrication, edge treatment and filter design. Design guidance is provided below in each of these areas.

1. General. Rock and wire mattress revetments can be constructed from commercially available wire units as illustrated in the details of Figures 38-6X and 38-6Y or from available wire fencing material as illustrated in Figure 38-6Z. The use of commercially available basket units is the most common practice and usually the least expensive.

   Rock and wire mattress revetments can be used to protect either the channel bank (as illustrated in the section of Figure 38-6X) or the entire channel perimeter (Figure 38-6Y). When used for bank protection, rock and wire mattress revetments consist of two distinct sections - a toe section and upper bank paving (see Figure 38-6X). As illustrated in Figure 38-6X, a variety of toe designs can be used. Emphasis in design should be placed on toe design and filter design. These designs are detailed later.

The vertical and longitudinal extent of the mattress should be based on guidelines provided in Section 38-5.07. Emphasis in design should be placed on toe design and filter design.

2. Bank and Foundation Preparation. Channel banks should be graded to a uniform slope. The graded surface, either on the slope or on the stream bed at the toe of the slope on which the rock and wire mattress is to be constructed, should not deviate from the specified slope line by more than 150 mm. All blunt or sharp objects (such as rocks or tree roots) protruding from the graded surface should be removed.

3. Mattress Unit Size and Configuration. Individual mattress units should be a size that is easily handled on site. Commercially available gabion units come in standard sizes as indicated in Figure 38-6AA. Manufacturer's literature indicates that alternative sizes can be manufactured when required, provided that the quantities involved are of a reasonable magnitude.

   The mattress should be divided into compartments so that failure of one section of the mattress will not cause loss of the entire mattress. Compartmentalization also adds to the structural integrity of individual gabion units. It is recommended that diaphragms be installed at a nominal 0.91-m spacing within each of the gabion units to provide the recommended compartmentalization (see Figure 38-6BB).

   On steep slopes ((greater than 1H:3V)), and in environments subject to high stresses (in areas prone to high flow velocities, debris flows, ice flows, etc.), diaphragms should be spaced at minimum intervals of 0.6 m to prevent movement of the stone inside the basket.

   The thickness of the mattress is determined by three factors - the erodibility of the bank soil, the maximum velocity of the water and the bank slope. The minimum thickness required for various conditions is tabulated in Figure 38-6CC. These values are based on observations of a large number of mattress installations which assume a filling material in the size range of 75 mm to 150 mm.

   The mattress thickness should be at least as thick as two overlapping layers of stone. The thickness of mattresses used as bank toe aprons should always exceed 300 mm. The typical range is 300 mm to 500 mm. The thickness of mattress revetments can vary according to need by utilizing gabions of different depths as illustrated in Figure 38-6X(d).

4. Stone Size. The maximum size of stone should not exceed the thickness of individual mattress units. The stone should be well graded within the sizes available, and 70 percent of the stone, by weight, should be slightly larger than the wire-mesh opening. For commercially available units, the wire-mesh opening sizes are listed in Figure 38-6AA.

Common median stone sizes used in mattress designs range from 75 mm to 150 mm for mattresses less than 0.3 m thick. For mattresses of larger thicknesses, rock having a median size up to 0.3 m is used.

5. Stone Quality. The stone should meet the quality requirements as specified for dumped-rock riprap.

6. Basket Fabrication. Commercially fabricated basket units are formed from galvanized steel wire mesh of triple twist hexagonal weave. The netting wire and binding wire is approximately 2.18 mm in diameter. The wire for edges and corners is approximately 2.69 mm in diameter. Manufacturer's instructions for field assembly of basket units should be followed.

   All wire used in the construction of the mesh rock enclosures (including tie wire) shall be zinc-coated (galvanized) to ASTM A 641M-92; the minimum weight of the zinc coating shall be based on Figure 38-6DD.

   Galvanized wire baskets may be safely used in fresh water and in areas where the pH of the liquid in contact with it is not greater than 10.

   For highly corrosive conditions, such as in salt water environments, industrial areas, polluted streams, and soils such as muck, peat and cinders, a polyvinyl chloride (PVC) coating must be used over the galvanizing. The PVC coating must have a nominal thickness of 0.5499 mm and shall nowhere be less than 0.381 mm. It shall be capable of resisting deleterious effects of natural weather exposure and immersion in salt water and shall not show any material difference in its initial characteristics with time.

7. Edge Treatment. The edges of rock and wire mattress revetment installations (the toe, head and flanks) require special treatment to prevent damage from undermining. Of primary concern is toe treatment. Figure 38-6X illustrates several possible toe configurations. If a toe apron is used, its projection should be 1.5 times the expected maximum depth of scour in the vicinity of the revetment toe. In areas where little toe scour is expected, the apron can be replaced by a single-course gabion toe wall which helps to support the revetment and prevent undermining. In cases where an excessive amount of toe scour is anticipated, both an apron and a toe wall can be used.

   To provide extra strength at the revetment flanks, it is recommended that mattress units having additional thickness be used at the upstream and downstream edges of the revetment (see Figure 38-6EE). It is further recommended that a thin layer of topsoil be spread over the flank units to form a soil layer to be seeded when the revetment installation is complete. The head of rock and wire mattress revetments can usually be terminated at grade as illustrated in Figure 38-6X.

8. Filter Design. Individual mattress units will act as a crude filter and a pavement unit when filled with overlapping layers of hand-size stones. However, it is recommended that the need for a filter be investigated. If necessary, a layer of permeable membrane cloth (geotextile) woven from synthetic fibers or a 100-mm to 150-mm layer of gravel should be placed between the silty bank and the rock and wire mattress revetment to further inhibit washout of fines.

9. Construction. Construction details for rock and wire mattresses vary with the design and purpose for which the protection is provided. Typical details are illustrated in Figures 38-6X, 38-6Y, and 38-6Z. Rock and wire mattress revetments may be fabricated where they are to be placed or at an off-site location. Fabrication at an offsite location requires that the individual mattress units be transported to the site; in this case extreme care must be taken so that moving and placing the baskets does not damage them by breaking or loosening strands of wire or ties or by removing any of the galvanizing or PVC coating. Because of the potential for damage to the wire enclosures, off-site fabrication is not recommended.

   Installation of mattress units above the water line is usually accomplished by placing individual units on the prepared bank, lacing them together, filling them with appropriately sized rock, and then lacing the tops to the individual units. A typical installation is illustrated in Figure 38-6FF. Where the mattress units must be placed below the water line in relatively shallow water, mattress units can be assembled at a convenient location and then be placed on the bank using a crane as illustrated in Figure 38-6GG. For deep water installations, an efficient method of large-scale placement is to fabricate the mattress sections on a barge or pontoon and then launch them into the water at the shore line (see Figure 38-6HH).

38-6.02(02) Design Guidelines for Stacked Block Gabions

Components of stacked gabion revetment design include layout of a general scheme or concept, bank and foundation preparation, unit size and configuration, stone size and quality, edge treatm-ent, backfill and filter considerations, and basket or rock enclosure fabrication. Design guidelines for stone size and quality and bank preparation are the same as those discussed for mattress designs; other remaining areas are discussed below.

1. General. Stacked gabion revetments are typically used instead of gabion mattress designs qwhere the slope to be protected is greater than 1H:1V or where the purpose of the

revetment is for flow training. Typical design schemes include flow training walls (Figure 38-6II(a)) and low retaining walls (Figure 38-6II(b)).

Stacked gabion revetments must be based on a firm foundation. The foundation or base elevation of the structure should be well below any anticipated scour depth. Additionally, in alluvial streams where channel bed fluctuations are common, an apron should be used as illustrated in Figure 38-6II(a) and (b). Aprons are also recommended for situations where the estimated scour depth is uncertain.

2. Size and Configuration. Common commercial sizes for stacked gabions are listed in Figure 38-6AA. The most common sizes used are those having widths and depths of 0.91 m. Sizes less than 0.3-m thick are not practical for stacked gabion installations.

Typical design configurations include flow training walls and structural retaining walls. The primary function of flow training walls (Figure 38-6II(a)) is to establish normal channel boundaries in rivers where erosion has created a wide channel or to realign the river when it is encroaching on an existing or proposed structure. A stepped-back wall is constructed at the desired bank location; counterforts are installed to tie the walls to the channel bank at regular intervals as illustrated. The counterforts are installed to form a structural tie between the training wall and the natural stream bank and to prevent overflow from scouring a channel behind the wall. Counterforts should be spaced to eliminate the development of eddy or other flow currents between the training wall and the bank which could cause further erosion of the bank. The dead water zones created by the counterforts so spaced will encourage sediment deposition behind the wall which will enhance the stabilizing characteristics of the wall.

Retaining walls can be designed in either a stepped-back configuration as illustrated in Figure 38-6II(b). Structural details and configurations can vary from site to site.

Gabion walls are gravity structures and their design follows standard engineering practice for retaining structures. Design procedures are available in standard soil mechanics texts as well as in gabion manufacturer's literature.

3. Edge Treatment. The flanks and toe of stacked block gabion revetments require special attention. The upstream and downstream flanks of these revetments should include counterforts; see Figure 38-6II(a). The counterforts should be placed 3.7 m to 5.5 m from the upstream and downstream limits of the structure and should extend a minimum of 3.7 m into the bank.

The toe of the revetment should be protected by placing the base of the gabion wall at a depth below anticipated scour depths. In areas where it is difficult to predict the depth of expected scour, or where channel bed fluctuations are common, it is recommended that a mattress apron be used. The minimum apron length should be equal to 1.5 times the anticipated scour depth below the apron. This length can be increased in proportion to the level of uncertainty in predicting the local toe scour depth.

4. Backfill/Filter Requirements. Standard gabion structure design requires the use of selected backfill behind the retaining structure to provide for drainage of the soil mass behind the wall. The permeable nature of gabion structures permits natural drainage of the supported embankment. However, because material leaching through the gabion wall can become trapped and cause plugging, it is recommended that a granular backfill material be used. The back fill should consist of a 50-mm to 300-mm layer of graded crushed stone backed by a layer of fine granular backfill.

5. Basket Fabrication. Commercially fabricated basket units are formed from galvanized steel wire mesh of triple twist hexagonal weave. The netting wire and binding wire specifications are the same as those discussed for mattress units. Specifications for galvanizing and PVC coatings are also the same for block designs as for mattresses. Figure 38-6JJ illustrates typical details of basket fabrication.

6. Construction. Construction details for gabion installations typically vary with the design and purpose for which the protection is being provided. Several typical design schematics were presented in Figures 38-6II and 38-6JJ.

   As with mattress designs, fabrication and filling of individual basket units can be done at the site or at an off-site location. The most common practice is to fabricate and fill individual gabions at the design site. The following steps outline the typical sequence used for installing a stacked gabion revetment or wall.

   a. Prepare the revetment foundation. This includes excavation for the foundation and revetment wall.

   b. Place the filter and gabion mattress (for designs which incorporate this component) on the prepared grade; then sequentially stack the gabion baskets to form the revetment system.

   c. Each basket is unfolded and assembled by lacing the edges together and the diaphragms to the sides.

   d. Fill the gabions to a depth of 0.3 m with stone from 100 mm to 300 mm in diameter. Place one connecting wire in each direction and loop it around two meshes of the gabion wall. Repeat this operation until the gabion is filled.

e. Wire adjoining gabions together by their vertical edges; stack empty gabions on the filled gabions and wire them at front and back.

f. After the gabion is filled, fold the top shut and wire it to the ends, sides and diaphragms.

g. Crushed stone and granular backfill should be placed at intervals to help support the wall structure. It is recommended that backfill be placed at three-course intervals.

38-6.03 Pre-Cast Concrete

Pre-cast concrete block revetments consist of pre-formed sections which interlock with each other, are attached to each other, or butt together to form a continuous blanket or mat. The concrete blocks which make up the mats differ in shape and method of articulation, but they share certain common features. These features include flexibility, rapid installation and provisions for the establishment of vegetation within the revetment.

38-6.03(01) Block Designs

Pre-cast concrete block designs come in a number of shapes and configurations. Figures 38-6KK, 38-6LL, 38-6MM, 38-6NN and 38-6 OO illustrate several commercially available concrete block designs. Note that other manufacturers and designs are available. Pre-cast revetments are bound using a variety of techniques. In some cases the individual blocks are bound to rectangular sheets of filter fabric (referred to as fabric carrier). Other manufacturers use a design which interlocks individual blocks. Other units are simply butted together at the site. The most common method is to join individual blocks with wire cable or synthetic fiber rope.

38-6.03(02) Design Guidelines

Components of a pre-cast concrete block revetment design include layout of a general scheme or concrete, bank preparation, mattress and block size, slope, edge treatment, filter design and surface treatment. Design information is provided below in each of these areas.

As illustrated in Figures 38-6KK, 38-6LL, 38-6MM, 38-6NN and 38-6 OO, pre-cast block revetments are placed on the channel bank as continuous mattresses. Emphasis in design should be placed on toe design, edge treatment and filter design.

1. <u>Bank Preparation</u>. Channel banks should be graded to a uniform slope. Any large boulders, roots and debris should be removed from the bank prior to final grading. Also, holes, soft areas and large cavities should be filled. The graded surface, either on the slope or on the stream bed at the toe of the slope on which the revetment is to be constructed, should be true to line and grade. Light compaction of the bank surface is recommended to provide a solid foundation for the mattress.

2. <u>Mattress And Block Size</u>. The overall mattress size is dictated by the longitudinal and vertical extent required of the revetment system. Articulated block mattresses are assembled in sections prior to placement on the bank; individual mattresses should be constructed to a size that is easily handled on site by available construction equipment. The size of individual blocks is quite variable from manufacturer to manufacturer. In addition, individual manufacturers usually have several standard sizes of a particular block available. Manufacturer's literature should be consulted when selecting an appropriate block size for a given hydraulic condition.

3. <u>Slope</u>. Articulated pre-cast block revetments can be used on bank slopes up to 1.5H:1V. However, an earth anchor should be used at the top of the revetment to secure the system against slippage (see Figures 38-6MM and 38-6NN). Pre-cast block revetments that are assembled by simply butting individual blocks end to end (with no physical connection) should not be used on slopes greater than 3H:1V.

4. <u>Edge Treatment</u>. The edges of pre-cast block revetments (the toe, head and flanks) require special treatment to prevent undermining. Of primary concern in the design of mattress revetments is the toe treatment. Two toe treatments have been used - an apron design, as illustrated in Figures 38-6KK and 38-6NN, and a toe trench design as illustrated in Figures 38-6LL and 38-6MM. As a minimum, toe aprons should extend 1.5 times the anticipated scour depth in the vicinity of the bank toe. If a toe trench is used, the mattress should extend to a depth greater than the anticipated scour depth in the vicinity of the bank toe.

   Two alternatives have also been used for edge treatments at the top and flanks. The edges can be terminated at-grade (Figures 38-6KK, 38-6LL and 38-6NN) or in a termination trench. Termination trenches are recommended in environments subject to significant erosion (silty/sandy soils and high velocities), or where failure of the revetment would result in significant economic loss. Termination trenches provide greater protection against failure from undermining and outflanking than do at-grade terminations. However, in instances where upper bank erosion or lateral outflanking is not expected to be a problem, grade terminations may provide an economic advantage.

For articulated designs, earth anchors should be placed at regular intervals along the top of the revetment (see Figures 38-6LL and 38-6MM). Anchors are spaced based on soil type, mat size and the size of the anchors. See manufacturer's literature for recommended spacings.

5. Filter. Prior to installing the mats, a geotextile filter fabric should be installed on the bank to prevent bank material from leaching through the openings in the mattress structure. Although a fabric filter is recommended, graded filter material can be used if it is properly designed and installed to prevent movement of the graded material through the protective mattress.

6. Surface Treatment. The spaces between and within individual blocks located above the low-water line should be filled with earth and seeded so that natural vegetation can be established on the bank (see Figures 38-6LL and 38-MM). This treatment enhances both the structural stability of the embankment and its aesthetic qualities.

38-6.03(03) Construction

Schematics of the types of pre-cast block revetments discussed above are provided in Figures 38-6KK, 38-6LL, 38-6MM, 38-6NN and 38-6OO. More detailed design sketches and information are available from individual manufacturers. Manufacturers also have available information on construction procedures. Some manufacturers will provide on-site advice and assistance in the installation of their systems.

Articulated pre-formed block revetments can be installed by construction crews using conventional construction equipment wherever a dragline or crane can be maneuvered. Construction procedures for most pre-formed block revetments are similar. After all site preparation work is completed, construction follows the following sequence.

1. Excavate toe, flank and upper bank protection trenches as required.

2. Place filter fabric and/or graded filter material on the prepared subgrade.

3. Individual mats are then attached to a spreader bar and lifted with a crane or backhoe for placement on the embankment slope. Mats are placed side by side on the bank until the entire prepared surface is covered.

4. Adjacent mats are secured to one another by fastening side connecting cables and end loops or by pouring side connecting keys.

5. Optional anchors are placed at the top and flanks of the protection as required.

6. Backfill is then spread over the mats (and into the open cells or spaces between cells) and into the anchor trenches. Anchor trenches are then compacted, and the general backfill should be seeded and fertilized according to local seasonal conditions.

Non-articulated block revetments (i.e., where the blocks are butted together instead of being physically attached) are constructed in a similar fashion, except that the individual blocks must be placed on the bank by hand, one at a time. This results in a much more labor-intensive installation procedure.

38-6.04 Grouted Riprap

38-6.04(01) Design Guidelines

Grouted revetment riprap consists of rock slope-protection having voids filled with concrete grout to form a monolithic armor. See Section 38-4.05 for additional descriptive information and general performance characteristics for grouted riprap.

Components of grouted rock riprap design include the layout of a general scheme or concept, bank preparation, bank slope, rock quality, grout quality, edge treatment, filter design and pressure relief.

Grouted riprap designs are rigid monolithic bank protection schemes. When complete, they form a continuous surface. A typical grouted riprap section is shown in Figure 38-6PP.

Grouted riprap should extend from below the anticipated channel bed scour depth to the design high- water level, plus additional height for freeboard.

During the design phase for a grouted riprap revetment, special attention needs to be paid to edge treatment, foundation design and mechanisms for hydrostatic pressure relief.

1. Bank And Foundation Preparation. The bank should be prepared by first clearing all trees and debris from the bank and grading the bank surface to the desired slope. In general, the graded surface should not deviate from the specified slope line by more than 150 mm. However, local depressions larger than this can be accommodated because initial placement of filter material and/or rock for the revetment will fill these depressions.

Because grouted riprap is rigid but not extremely strong, support by the embankment must be maintained. To form a firm foundation, it is recommended that the bank surface be tamped or lightly compacted. Care must be taken during bank compaction to maintain soil permeability similar to that of the natural, undisturbed bank material. The foundation for the grouted riprap revetment should have a bearing capacity sufficient to support either the dry weight of the revetment alone or the submerged weight of the revetment plus the weight of the water in the wedge above the revetment for design conditions, whichever is greater.

2. Bank Slope. Bank slopes for grouted riprap revetments should not exceed 1.5H:1V.

3. Rock Quality. Rock used in grouted rock slope-protection is usually the same as that used in ordinary rock slope-protection. However, the specifications for specific gravity and hardness may be lowered if necessary as the rocks are protected by the surrounding grout. In addition, the rock used in grouted riprap installations should be free of fines in order that penetration of grout may be achieved.

4. Grout Quality And Characteristics. Grout should consist of good strength concrete using a maximum aggregate size of 19 mm and a slump of 75 mm to 100 mm. Sand mixes may be used where roughness of the grout surface is unnecessary, provided sufficient cement is added to give good strength and workability.

   The finished grout should leave face stones exposed for one-fourth to one-third their depth and the surface of the grout should expose a matrix of coarse aggregate. See Section 904 of the *Standard Specifications* for more information on grout.

5. Edge Treatment. The edges of grouted rock revetments (the head, toe and flanks) require special treatment to prevent undermining. The revetment toe should extend to a depth below anticipated scour depths or to bedrock. The toe should be designed as illustrated in Figure 38-6PP(a). After excavating to the desired depth, the riprap slope protection should be extended to the bottom of the trench and grouted. The remainder of the excavated area in the toe trench should be filled with ungrouted riprap. The grout-free riprap provides extra protection against undermining at the bank toe.

   To prevent outflanking of the revetment, various edge treatments are required. Recommended designs for these edge treatments are illustrated in Figure 38-6PP (a), (b) and (c).

6. Filter Design. Filters are required under all grouted riprap revetments to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressures. A 150-mm granular filter is required beneath the pavement to provide an adequate drainage zone. The filter can consist of well-graded

granular material or uniformly-graded granular material with an underlying filter fabric. The filter should be designed to provide a high degree of permeability while preventing base material particles from penetrating the filter, thus causing clogging and failure of the protective filter layer.

7. Pressure Relief. Weep holes should be provided in the revetment to relieve hydrostatic pressure build-up behind the grout surface (see Figure 38-6PP(a)). Weeps should extend through the grout surface to the interface with the gravel underdrain layer. Weeps should consist of 75-mm diameter pipes having a maximum horizontal spacing of 1.8 m and a maximum vertical spacing of 3.0 m. The buried end of the weep hole should be covered with wire screening or a fabric filter of a gage that will prevent passage of the gravel underlayer.

38-6.04(02) Construction

Construction details for grouted riprap revetments are illustrated in Figure 38-6PP. The following construction procedures should be followed.

1. Normal construction procedures include (a) bank clearing and grading; (b) development of foundations; (c) placement of the rock slope protection; (d) grouting of the interstices; (e) back- filling toe and flank trenches; and (f) vegetation of disturbed areas.

2. The rock should be wet immediately prior to commencing the grouting operation.

3. The grout may be transported to the place of final deposit by chutes, tubes, buckets, pneumatic equipment or any other mechanical method which will control segregation and uniformity of the grout.

4. Spading and rodding are necessary where penetration is achieved by gravity flow into the interstices.

5. No loads should be allowed upon the revetment until good strength has been developed.

38-6.05 Grouted Fabric Slope Paving

Grouted fabric-formed revetments are a relatively new development for use on earth surfaces subject to erosion. They have been used as an alternative to traditional revetments such as concrete liners or riprap on reservoirs, canals and dikes.

Grouted fabric-formed revetments are made by pumping a highly fluid structural grout, often referred to as “fine-aggregate concrete,” into an insitu envelope consisting of a double-layer synthetic fabric. During filling, excess mixing water is squeezed out through the highly permeable fabric substantially reducing the water/cement ratio with consequent improvement in the quality of the hardened concrete. A major advantage of this type revetment is that fabric-formed revetments may be as easily assembled underwater as in a dry location.

38-6.05(01) Types

The three commonly used types of fabric-formed revetments are as follows:

1. Type 1. Two layers of nylon fabric are woven together at 125-mm to 250-mm centers as indicated in Figure 38-6QQ. These points of attachment serve as filter points to relieve hydrostatic uplift caused by percolation of ground water through the underlying soil. The finished revetment has a deeply cobbled or quilted appearance. Mat thicknesses typically average from 50 mm to 150 mm.

2. Type 2. Two layers of nylon or polypropylene woven fabric are joined together at spaced centers by means of interwoven tie cords, the length of which controls the thickness of the finished revetment. See Figure 38-6RR. Plastic tubes may be inserted through the two layers of fabric prior to grout injection to provide weep holes for relief of hydrostatic uplift. The finished revetment is of relatively uniform cross section and has a lightly pebbled appearance. Mat thicknesses typically average from 50 mm to 250 mm.

3. Type 3. Two layers of nylon fabric are interwoven in a variety of rectangular block patterns, the points of interweaving serving as hinges to permit articulation of the hardened concrete blocks. Revetments are reinforced by steel cables or nylon rope threaded between the two layers of fabric prior to grout injection and remain embedded in the hardened cast-in-place blocks. Block thickness is controlled by spacer cords in the middle of each block.

38-6.05(02) Design Guidelines

The specially woven fabric for grouted fabric-formed revetments are manufactured by several different companies. The designer should consult the manufacturer’s literature for designing and selecting the appropriate type of material and thickness for a given hydraulic condition.

38-6.06 Sand-Cement Bags

Sand-cement bags generally consist of a dry mix of sand and cement placed in a burlap or other suitable bag. They require firm support from the protected bank. Usually a filter fabric is placed underneath this type of riprap. Adequate protection of the terminals and toe is essential. The riprap has little flexibility and low tensile strength, and it is susceptible to damage particularly on flatter slopes where the area of contact between the bags is less.

38-6.06(01) Design Guidelines

Concrete riprap in bags generally consists of approximately 0.0189 m^3 of class C concrete (4.6 bags cement/m^3) in a burlap bag or in a cement sack. Each bag should be tied if in cement sacks or the top folded around the bag if in burlap sacks. This type of riprap provides a heavy protection regardless of the requirements of the site. The riprap has little flexibility and low tensile strength, and it is susceptible to damage from floating ice. It requires firm support from the protected bank and usually requires a filter blanket underneath the riprap. Adequate protection of the terminals and toe is essential. The toe trench must end in firm support and extend below the depth of anticipated scour. Details of terminal protection and cutoff stubs are shown in Figure 38-6SS.

The bags make close contact with each other and some bond is secured between the bags by the cement mortar leaking through the porous bags. Flat slopes reduce the area of contact between the sacks and thus the bond is less. Slopes of the protected embankment are generally 1.5H:1V. If the slopes are as flat as 2H:1V, all sacks after the bottom row should be laid as headers (long way of sack in line with the slope) rather than as stretchers (long way at right angles to slope direction).

Concrete riprap in bags is sometimes placed as a dry mix. The riprap is thoroughly wetted as the work progresses. Some bond between sacks is probably lost by this method, but it allows the sacks to be filled at a convenient location and brought to the construction site. A well-graded filter blanket is essential to drain the water that is added during construction.

38-6.06(02) Construction

Cloth cement sacks about two-thirds filled and securely tied or burlap grain sacks containing about 0.0189 m^3 of concrete and folded at the top are immediately placed in position after filling. The fold on burlap bags is placed underneath the bag for headers and against the previously placed sack for stretchers. Where the protected slope is 1.5H:1V or steeper, a bed consisting of two rows of sacks placed as stretchers is followed by a row of sacks placed as headers. Succeeding rows of sacks are placed as stretchers with joints between sacks staggered (see Figure 38-6SS). Each sack is hand placed and pushed into firm contact with adjacent sacks. On slopes flatter than 1.5H:1V all rows after the bed row are placed as headers.

Cutoffs and weep holes shall be placed as shown on the plans or as directed by the designer. The finished work should present a neat appearance with parallel rows of sacks, and no sack shall protrude more than 75 mm from the finished surface.

The riprap should be placed only when the temperature is above 2^{o}C and rising. It should be protected from freezing and cured as for concrete.

Whenever placement of concrete riprap in bags is delayed sufficiently to affect the bond between succeeding courses, a small trench about half the depth of sack should be excavated back of the last row of sacks in place and the trench filled with fresh concrete before the next layer of sacks is laid. At the start of each day's work or when a delay of over 2 hours occurs during the placing of successive layers of sacks, the previously placed sacks should be moistened and dusted with cement to develop bond.

38-6.07 Soil-Cement

Soil-cement is an acceptable method of slope protection for dams, dikes, levees, channels and highway embankments. Soil-cement can also be used to construct impervious cores in retention dam type structures and provide a protective facing. On most projects, soil-cement is constructed in stair-step fashion by placing and compacting the soil-cement in horizontal layers stair-stepped up the embankment (See Figure 38-6TT). This facilitates placement using common highway construction equipment. Embankment slopes of from 2.5H:1V and 4H:1V and horizontal layer widths of from 2.13 m to 2.74 m provide minimum protective thicknesses of about 0.46 m to 0.76 m measured normal to the slope.

A wide variety of soils can be used to make durable soil-cement slope protection. The Portland Cement Association (PCA) has data on soil types, gradations, costs and testing procedures. The PCA also has data on placement and compaction methods.

Use of soil-cement does not require any unusual design considerations for the embankment. Proper embankment design procedures should be followed, based on individual project conditions, to prevent subsidence or any other type of embankment distress.

38-6.07(01) Design Guidelines

1. Top, Toe and End Features. An important consideration in the design of soil-cement facing is to ensure that all extremities of the facing are tied into nonerodible sections or abutments. Adequate freeboard and carrying the soil-cement to the paved roadway, plus a lower-section detail as shown in Figure 38-6TT, will minimize erosion from behind the crest and under the toe of the facing. The ends of the facing should terminate smoothly in flat slopes or against rocky abutments. In some installations, a small amount of rock riprap may be placed over and adjacent to the edges of the soil-cement at its contact with the abutments.

   Where miscellaneous structures such as culverts extend through the facing, the areas immediately adjacent to such structures are constructed by placing and compacting the soil-cement by hand or with small power tools or by using a lean-mix concrete.

2. Special Conditions. Slope stability is provided to embankments by the strength and impermeability of the soil-cement facing. Special design considerations usually are not necessary in soil-cement-faced embankments. It is necessary to utilize proper design and analysis procedures to ensure the structural and hydraulic integrity of the embankment. Conditions most commonly requiring special analysis include subsidence of the embankment or rapid drawdown of the reservoir or river.

3. Subsidence. Embankment subsidence results from a compressible foundation, settlement within the embankment itself or both. Analyzing the possible effects of such a condition involves a number of assumptions by the designer concerning the embankment behavior. Combining these assumptions with the characteristics of the facing, a structural analysis of the condition can be made. If the unit weight and flexural strength of the soil-cement are not known, they can be taken as 1924 kg/m^3 and 1.034 - 1.379 MPa, respectively. The layer effect can usually be ignored.

   Note: The post construction appearance of a pattern of narrow surface cracks about 3.0 m to 6.0 m apart is evidence of normal hardening of the soil-cement. Substantial embankment subsidence conceivably could allow the facing to settle back in large sections coinciding with the normal shrinkage crack pattern. If such settlement of the soil-cement, with separation at the shrinkable cracks, takes place, the slope remains adequately protected unless the settlement is large enough to allow the outer face of a settling section to move past the inner face of an adjoining section.

4. Rapid Drawdown. Rapid drawdown exceeding 4.6 m or more within a few days theoretically produces hydrostatic pressure from moisture trapped in the embankment against the back of the facing. Three design concepts that may be used to prevent damage due to rapid drawdown-induced pressure are as follows:

a. designing the embankment so that its least permeable zone is immediately adjacent to the soil-cement facing, which ensures that seepage through cracks in the facing will not build up a pool of water sufficient to produce damaging hydrostatic pressure;

b. arbitrarily assuming the weight of the facing sufficient to resist any uplift pressures that may develop; and

c. providing free drainage behind, through or under the soil-cement facing to prevent adverse hydrostatic pressure.

38-6.07(02) Construction

The method of construction (central plant or mixed in place) should be considered by the designer in determining the facing cross section. Both methods have been successfully used for soil-cement slope protection. The central plant method, however, allows faster production and provides maximum control of mixing operations. With the mixed-in-place method, mixing should be deep enough so that there will be no unmixed seams between the layers, but excessive striking of the soil-cement below the layer being mixed should be avoided. A compacted layer thickness of 150 mm is most widely used, with the recommended maximum being 230 mm for efficient, uniform compaction.

The central-plant method should be more economical for all but the smallest projects, but it is advisable to allow the contractor the option of using either method where the quantity of soil-cement involved is only a few thousand cubic meters. The PCA has sample specifications regarding these two construction methods.

38-7.0 REFERENCES

1. U.S. Corps of Engineers, April 1985, *Design of Coastal Revetments, Seawalls, and Bulkheads*, Engineering Manual EM-1110-2-1614.

2. U.S. Department of Transportation, Federal Highway Administration, 1989, *Design of Riprap Revetment*, Hydraulic Engineering Circular No. 11.

3. U.S. Department of Transportation, Federal Highway Administration, 1995, *Geosynthetic Design and Construction Guidelines*, FHWA-HI-95-038.

4. U.S. Department of Transportation, Federal Highway Administration, February 1991, *Stream Stability at Highway Structures*, Hydraulic Engineering Circular No. 20.

5. Holtz, Robert D., Christopher, Barry R. and Berg, Ryan R., *Geosynthetic Design & Construction Guidelines*, Participant Notebook, July 29, 1994 Draft, National Highway Institute, US Department of Transportation, Federal Highway Administration.